MIL07

Via Lombardia 1, Siziano, MI, Italy

STACK INFRASTRUCTURE

Report - Civil - Stormwater Management

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1. SCOPE OF WORK

The purpose of this document is to describe the design and verification of the Stormwater Management works for the project in Via Liguria, Siziano (PV).

The project involves the construction of a building intended for productive activities such as the installation and management of IT infrastructures - Data Center.

This document constitutes the preliminary study for defining the stormwater management strategy and does not represent the stormwater management project in accordance with R.R. no.7 of 23 November 2017 of Lombardy Region as amended and supplemented.

The sizing of the project's stormwater management networks and the first-flush treatment works of the areas subject to R.R. no. 4 of 24 March 2006 of Lombardy Region are detailed in the dedicated document MIL07-DBA-EX-XX-CA-C-52001.

The project team addressed the stormwater management systems design in an integrated manner from the earliest design steps, enabling the application of sustainable drainage solutions, that comply with the requirements of European policies, as well as national and regional regulations regarding hydraulic risk, sustainability and adaptation and mitigation to climate change.

The project team approached the design of stormwater management systems in an integrated manner from the earliest stages, enabling the implementation of sustainable drainage solutions that comply with European policies as well as national and regional regulations on hydraulic risk, sustainability, and climate change adaptation and mitigation.

A multi-criteria analysis was carried out prior to the design phase of these systems, considering the following factors:

- possible infiltration constraints from the Municipal Government Plan,
- site topography,
- creation of a balanced master plan in terms of green and impermeable areas,
- assessment of areas at risk of spillage or subject to the first-flush regulation;
- assessment of the interference with threshold concentrations of soil contamination.

The project's stormwater management system incorporates both traditional systems for collecting water (manholes, pipes and concrete tanks) and green infrastructure i.e. NBS/Sustainable Drainage Systems (SuDS).

Wherever possible, green infrastructure will be used; where one of the constraints listed above inhibits its realisation, traditional infrastructure will be built.

The calculation of the flood storage volume, used to size hydraulic resilience measures, was carried out based on the duration maximizing reservoir capacity, comparing single intense events lasting 1-24 hours for each return period described in the following paragraphs.

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2. DESIGN STRATEGY

The site in the design state, features a significant amount of covered and impervious surface area, and the intervention is characterized by high complexity and scale.

To mitigate direct and indirect biophysical systems (heat island, pollution, etc.) with economically and environmentally sustainable measures, the implementation of innovative and multifunctional stormwater management solutions consisting of green infrastructure/NBS (Nature Based Solutions) if is possible, accompanied by a LID (Low Impact Development) design approach is proposed.

The combination of these strategies promotes the local detention of meteoric flows upstream of the drainage networks and infiltration by exploiting the retentive and purifying capacities of the soil-vegetation binomial. This optimizes the drainage network and significantly reduces the volumes to be stored in primary flow attenuation structures. These infrastructures are multifunctional, generating economic benefits, increasing biodiversity, and improving the well-being of people and their environment.

The proposed stormwater management system is based on the following principles:

- **Integration**: an integrated approach across various disciplines (engineering, architecture, landscape, agriculture, logistics, plant engineering, etc.) ensures that the concept design aligns with the development objectives and goals.
- **Distribution**: Balancing permeable and impermeable areas and dispersing water to avoid concentration help increase stormwater runoff time. Hydraulically independent sub-basins allow for easier flood peak management and reduced impacts.
- **Resilience**: Robust flood peak control and detention volume reduction are complemented by frequent infiltration, filtration, and treatment actions, creating a flexible system capable of absorbing variations.
- **Redundancy**: Independent sub-basin interventions allow for effective distribution and containment of unexpected events.

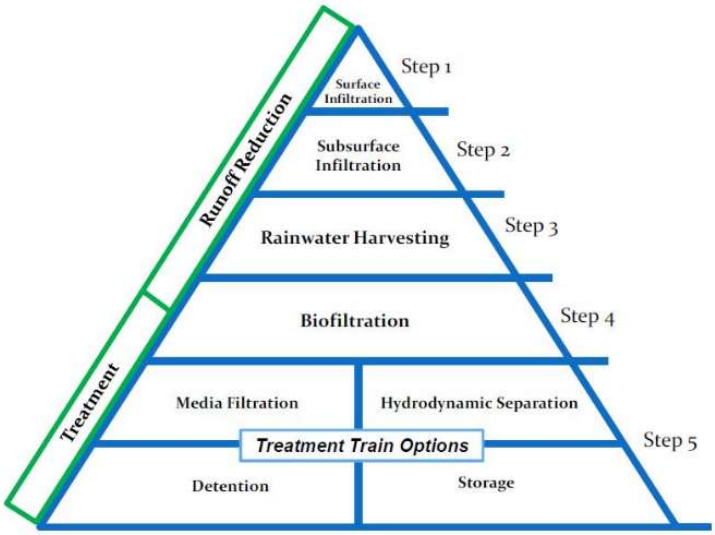

Figure 2.1: Five-stage LID technology

At the regional level, EU directives have been incorporated into Regional Regulation 7/2017 and subsequent amendments. According to this regulation, the control and management of rainwater must prioritize systems that ensure infiltration, evapotranspiration, and reuse where possible.

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The disposal of rainwater volumes follows this hierarchy:

- **Reuse**: Use stored rainwater for activities such as watering gardens, washing pavements, and cars, provided it meets quality standards and is feasible.
- **Infiltration**: Infiltrate rainwater into the soil or the superficial layers of the subsoil, ensuring compatibility with the soil's pedological characteristics, the subsoil's hydrogeological features, environmental and health regulations, and the guidelines in the geological, hydrogeological, and seismic components of the Municipal Government Plan.
- **Surface Water Discharge**: Discharge rainwater into natural or artificial surface water bodies, adhering to the flow limits set by regional regulations.
- **Sewerage System Discharge**: Discharge rainwater into the sewerage system, within the flow rate limits specified in regional regulations.

The methods listed above represent sustainable stormwater management practices aligned with EU directives.

Below is an illustrative image of a Low Impact Development (LID) approach incorporating sustainable drainage systems, taken from "Low Impact Development Opportunities for the Planet Region".

Figure 2.2: Innovative stormwater management

The project is designed to create balanced and integrated spaces in which architecture, landscape, habitat and stormwater management systems coexist and cooperate, mitigating both direct and indirect biophysical impacts and reducing hydrogeological risk. This approach enhances sustainability and resilience.

The project presents many constraints from an environmental point of view, which are discussed in detail in chapter 3, the most relevant of which is low water table depth. For this reason, it was decided to use underground concrete tanks to hold impermeable and covered surfaces.

The project includes separate collection networks for runoff water from the yards, which undergo dedicated pre-treatment for sand and oil removal, and rainwater from downspouts. For detailed sizing information, refer to the report MIL07-DBA-EX-XX-CA-C-52001. Both types of water are directed towards concrete underground detention tanks.

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To optimize the drainage network – minimizing pipe runs, diameters, and excavation depths – the layout adopts two underground detention tanks (**DT1** and **DT2**). DT1, located in the south-west sector of the site, collects runoff from streets and yards in the north-west, west, and south areas, as well as roof flows from downspouts along the main building's south façade. DT2, located in the south-east sector, collects runoff from streets and yards in the north-east and east areas, roof flows from downspouts along the north façade, and runoff from areas subject to the regional first-flush regulation (RR 04/2006).

For the sizing of these works please refer to Chapter 6.

To make the green areas hydraulically independent, they will be kerbed and locally lowered to prevent runoff from draining into the site. A detention pond (DP1) is planned within the internal green area.

In the subsequent design phase, the inclusion of infiltration works at the bottom of DP1 will be assessed. Once the existing and design elevations of the external perimeter green areas are defined, the need for additional detention basins or other NBS will also be evaluated.

The positioning and preliminary sizing of DP1 will be constrained by:

- the permeability of the soils forming the project embankment;
- any environmental constraints related to soil quality;
- interference with planned underground utilities;
- coordination with the landscape design;
- potential interference with the groundwater table.

These same constraints will also apply to any additional detention ponds or NBS that may be designed in the subsequent phase.

DP1 will be sized to ensure the draining green internal area remain hydraulically independent for the reference design period (for the selection of which, see Chapter □), considering only the infiltrating flow rate as the outflow. For sizing criteria and calculations, refer to Chapter 6.

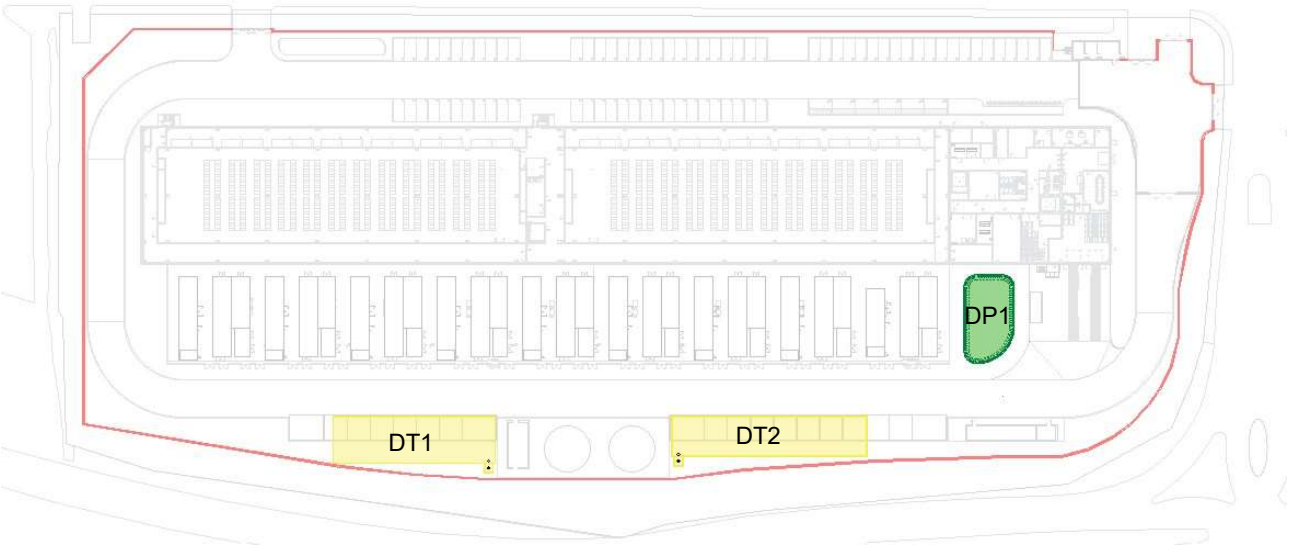

Figure 2.3: Layouts plan of the detention tanks DT1 and DT2 and the detention pond DP1

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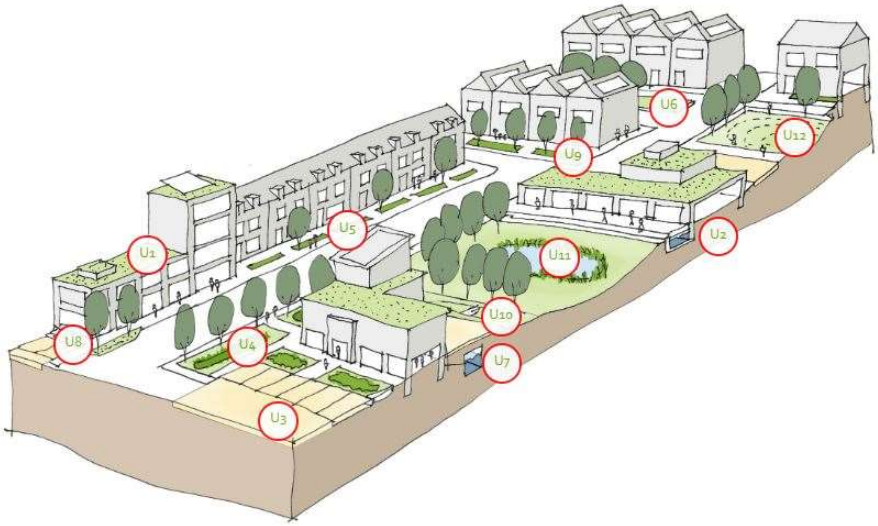
2.1. General functions of SuDS

This section outlines the design solutions adopted, detailing their functionality and materials.

Sustainable Drainage Systems (SuDS) have been thoroughly addressed in international and, more recently, national guidelines. Figure 2.4 presents an example application in an urban context, while Figure 2.5 schematically illustrates the main types of SuDS. These images are sourced from the NWRM pilot project funded by the European Commission's Directorate-General for the Environment.



U01	Green Roofs
U02	Rainwater Harvesting
U03	Permeable surfaces
U04	Swales
U05	Channels and rills
U06	Filter Strips
U07	Soakaways
U08	Infiltration Trenches
U09	Rain Gardens
U10	Detention Basins
U11	Retention Ponds
U12	Infiltration basins

Figure 2.4: Examples of SuDS applied in an urbanized context

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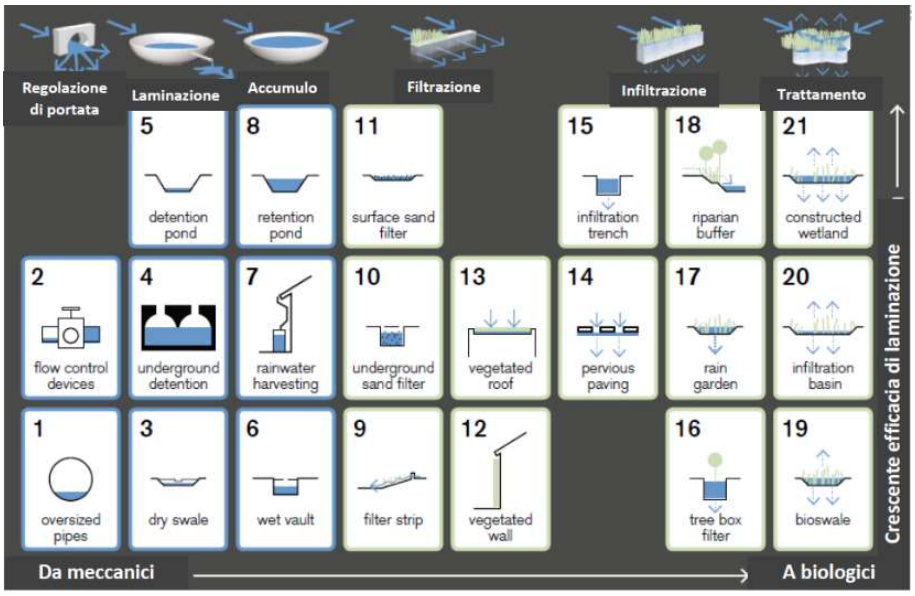

Figure 2.5: Schematization of the main sustainable drainage systems

Starting from the analysis of typical SuDS reported in technical literature, extensive and multifunctional mixed works have been designed to fulfill multiple and complementary purposes:

1. allow large, distributed volumes of detention/retention;
2. increase infiltration to consequently reduce the attenuation volume upstream;
3. install components in the stormwater management system with low construction costs and, in LCA terms, with low environmental impacts and low material and energy consumption;
4. implement certified native herbaceous, shrub and tree vegetation;
5. achieve high aesthetic and ecological value;
6. improve the usability of the area;
7. reduce of the heat island.

2.2. The role of soil

Soil is the first and most important element in building a sustainable drainage system. It is an ecologically and economically valuable, limited, and non-renewable resource.

The agronomic characteristics of the topsoil can significantly impact the system's effectiveness, potentially reducing field capacity, hindering vegetation growth, reducing evapotranspiration capacity, and increasing infiltration rates.

The table below, taken from the literature, shows the possible effects of varying soil characteristics.

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TABELLA 2 - Principali caratteristiche del terreno in funzione della tessitura

Parametro	Argilloso	Limoso	Sabbioso	Medio impasto
Csc (1)	alta > 20	media 10-20	bassa < 10	medio alta
Ritenzione idrica	alta	media	scarsa	media-alta
Mobilità elementi nutritivi	scarsa	scarsa	alta	media
Tendenza alla formazione di crepe	alta	media	scarsa	scarsa
Ristagno idrico	alta	alta	bassa	media
Mineralizzazione sostanza organica	bassa	bassa	alta	media
Tendenza alla formazione di crosta	bassa	alta	bassa	bassa

(1) Capacità di scambio cationico.

Figure 2.6: Main soil characteristics as a function of texture

Wherever SuDS are planned in the project layout, appropriate soil preparation will be carried out through aeration. If the existing topsoil is unsuitable, a draining substrate will be recreated to increase temporary water retention.

In cases where soil removal and replacement are necessary, the following minimum characteristics are recommended:

- 50-60% sand;
- 20-30% topsoil;
- 20-30% compost.

These values are indicative and should be confirmed during the execution phase in consultation with the appointed agronomist.

2.3. Types of suds used

The SuDS typologies used in the project are as follows:

- depressed green areas;
- semi-permeable paving and surfaces;
- detention and infiltration basins.

2.3.1. Depressed green areas

All green areas will be lowerd to provide additional attenuation volume and promote widespread infiltration. This design allows for drainage within the green area itself and neighbouring areas.

Typically, the depression between green areas and adjacent impermeable surfaces will range from 15 to 30 cm.

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Figure 2.7: Typical depressed green area

2.3.2. Semi-permeable paving and surfaces

Permeable paving is suitable for pedestrian and vehicular areas, accommodating light to heavy traffic.

This type of surfacing, combined with an appropriate drainage subbase, allows for partial water containment, filtration, and infiltration.

This approach facilitates diffuse rainwater management prior to concentration, with additional treatment occurring through subsoil filtration.

These surfaces perform several functions:

- filtration,
- adsorption,
- biodegradation,
- sedimentation.

Permeable pavements can be either continuous or discontinuous.

Continuous pavements resemble conventional road surfaces but use permeable asphalt or concrete mixes, achieved by reducing sand and other fine aggregates in the mix.

Draining asphalt pavements feature a permeable wearing course over an impermeable binder, ensuring effective surface drainage and filtration. However, to match the benefits of other types, they require integration with different sustainable drainage systems.

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Discontinuous pavements, on the other hand, involve prefabricated concrete elements that are perforated and interlocking.

Figure 2.8: Examples of continuous and discontinuous permeable pavements

2.3.3. Detention and infiltration Basins

Detention areas serve to temporarily store rainwater during heavy rainfall, gradually releasing it afterward to reduce strain on urban drainage networks. These systems employ various drainage methods, including point infiltration systems (such as soak pits and trenches) and extensive systems like infiltration basins.

In our project, we have incorporated one detention and infiltration basin named DP1 illustrated in Figure 2.3. To enhance infiltration efficiency, the basins should be situated on soils with high permeability, ideally sandy soils with coarse gravel that promote drainage and prevent water pooling. Infiltration basins typically feature grass cover on their banks and bottom to stabilize the area and filter pollutants carried by rainwater. Plant roots further enhance soil infiltration capacity by creating preferential infiltration pathways.

The depth of the basin is calculated to maintain a maximum water retention time of 24-48 hours during hydraulic sizing events. For managing flooding during shorter return periods (e.g., 2 or 5 years), additional measures like drainage trenches or underground attenuation solutions can be integrated to minimise visible surface impacts.

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Figure 2.9: Example of a detention and infiltration basin

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3. ANALYSIS OF HYDRAULIC CONSTRAINTS FOR INFILTRATION AND DETENTION WORKS

The implementation of detention and infiltration systems necessitates a thorough analysis of numerous environmental and territorial constraints that can restrict their deployment and impact their effectiveness. These constraints encompass various aspects, including:

- territorial limitations, such as compliance with strategic infrastructure buffer zones (e.g., drinking water wells, power lines, methane pipelines);
- operational effects of large hydraulic structures that may alter groundwater table levels;
- environmental considerations like soil and groundwater contamination;
- site-specific factors affecting soil infiltration capacity, such as groundwater levels, presence of impermeable soils, and underground utilities or constructions.

For detailed insights and comprehensive analyses concerning constraint and environmental considerations, please refer to specialized documents dedicated to these topics.

Below is a summary of typical hydraulic design constraints:

- topographical analysis;
- piezometric assessments in the intervention area:
- potential buffer strip requirements around drinking water wells:
- hydraulic permeability assessments of soils.

These factors play a critical role in determining the feasibility and design specifics of detention and infiltration systems within specific project contexts.

3.1. Topographical analysis

The site lies in the northern sector of the Municipality of Siziano. A topographic survey is available for the internal areas of the site and the surrounding streets, while the external perimeter green areas have not yet been surveyed. Based on the available data, the site platform currently sits lower (approximately 91.8 m a.s.l.) than the adjacent roads (approximately 92.0 m a.s.l.).

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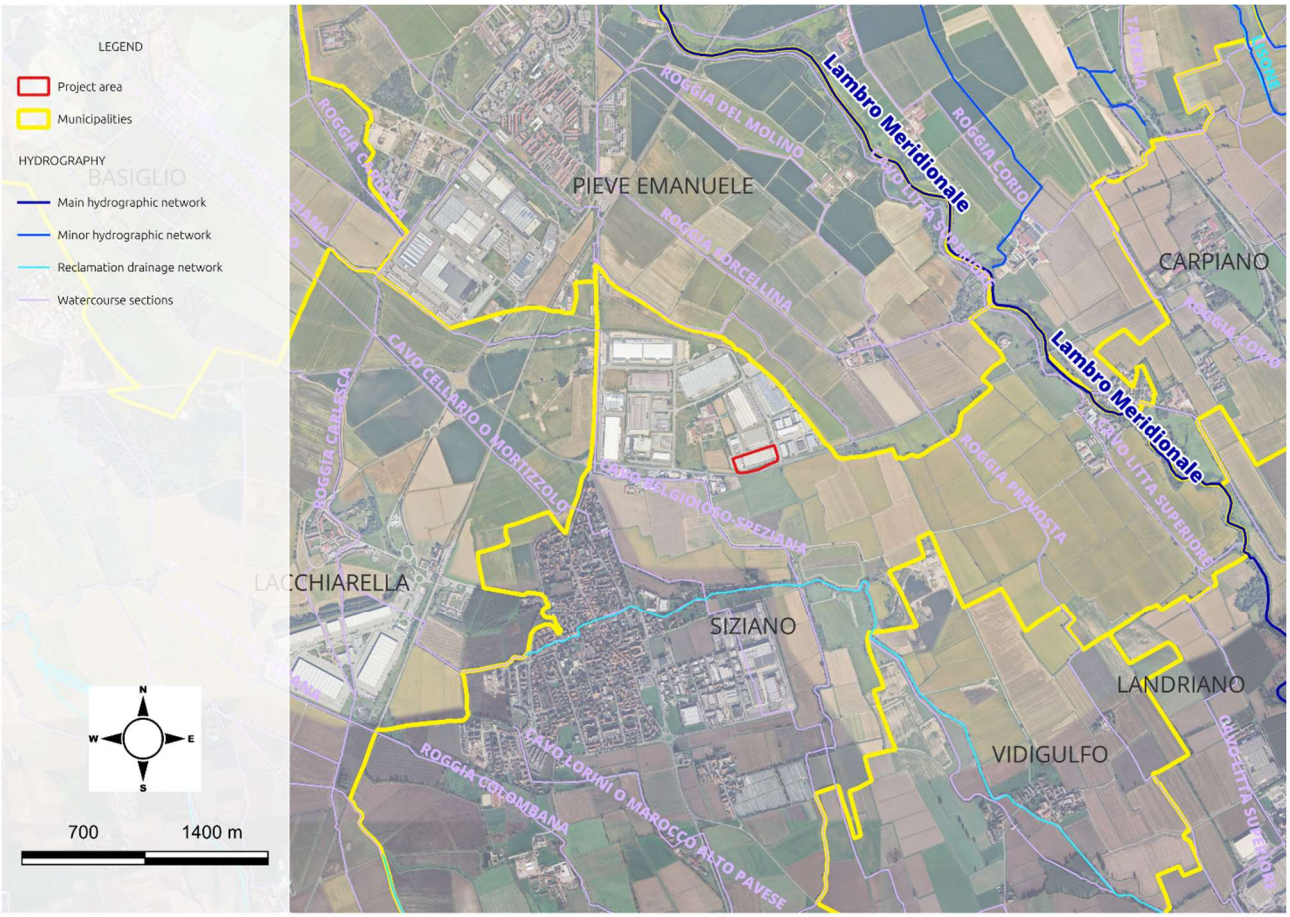

Figure 3.1: general site overview

Although the project grading has not been finalized, and in agreement with the Client, the preliminary design assumes a flat platform across the entire site at elevation 92.0 m a.s.l., approximately matching the road level along Via Liguria (north boundary).

For completeness, the regional 5x5 Digital Terrain Model (DTM) – noting that the project area is not covered by the 1×1 m LiDAR – was also consulted. This analysis indicates a general decrease in elevation from northwest to southeast, draining towards the Lambro Meridionale.

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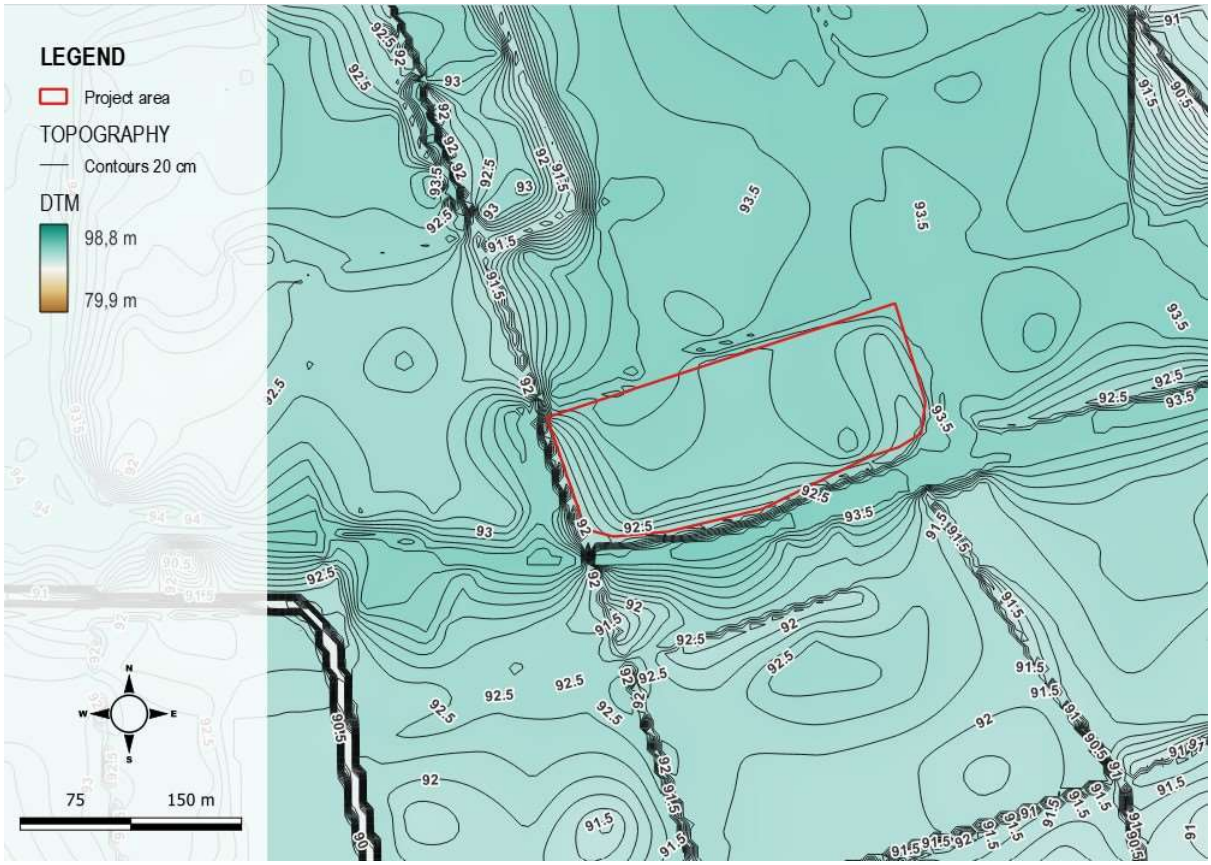

Figure 3.2: Topographic framework from DTM Lombardia 5x5 m

3.2. Piezometry

As shown in the cartographic extracts from the Plan for the Government of the Territory (PGT) of the Municipality of Siziano, the site has a water table level of approximately 88 and 89 m a.s.l.

Considering the existing average ground elevation of 91.8 m a.s.l., the current depth to groundwater is therefore about 2.8–3.8 m below ground level. Under the design assumption of a flat platform at 92.0 m a.s.l., the expected depth to groundwater would range between approximately 3.0 and 4.0 m below ground level, in line with the PGT indications.

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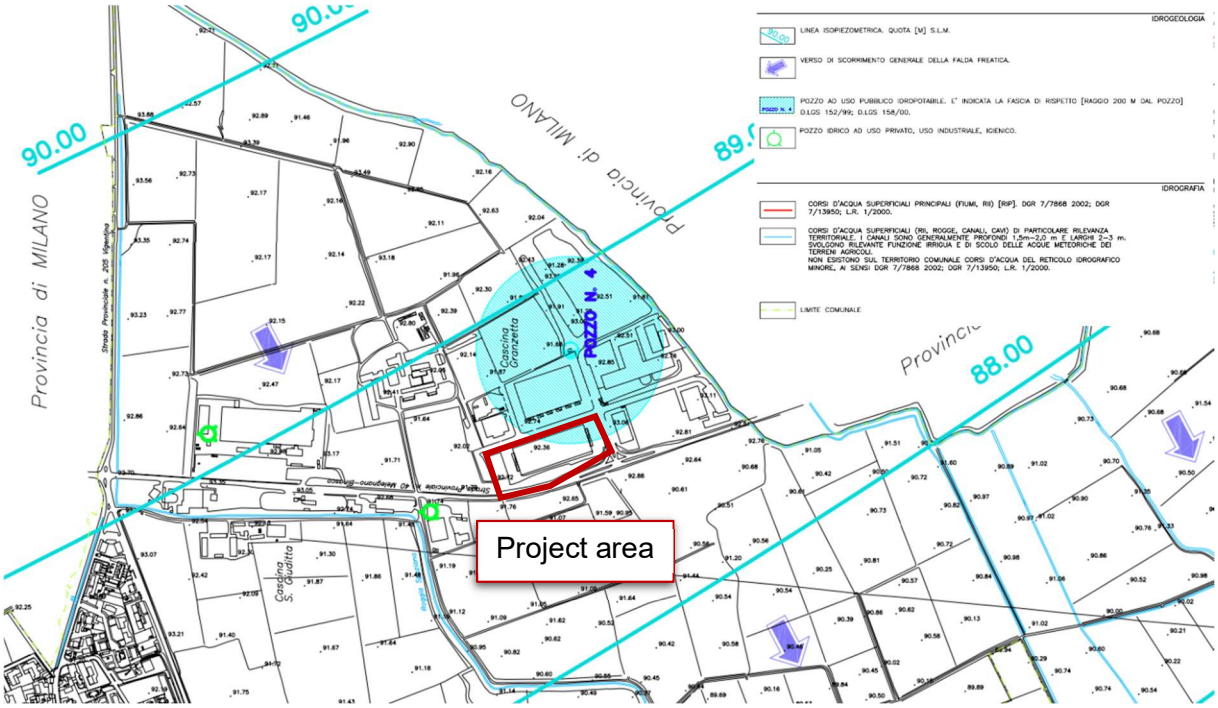

Figure 3.3: Hydrogeological map and hydrographic network map Tav.2 – PGT Siziano

During the geognostic investigation program, n.3 continuous coring surveys were carried out to a depth of 30.0 m from the current ground level (S1, S2, PZ1), and n.3 additional continuous coring surveys were carried out to a depth of 10.0 m from the current ground level (PZ2, PZ3, PZ4).

The investigations were conducted on July 21 and 22, 2025, by geologist S. Scarabelli and were documented in a dedicated report.

A map showing the location of the boreholes is provided in Figure 3.4, while Table 3.1 reports the groundwater level values recorded at each borehole.

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Figure 3.4: Orthophoto with the location of geognostic investigations

Table 3.1: Results of continuous coring surveys

SURVEY NUMBER	GROUNDWATER DEPTH [m]
S1	1.1
S2	2.3
PZ1	1.7
PZ2	2.4
PZ3	2.4
PZ4	2.4

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3.3. Verification of Buffer Zones from Potable Water Wells According to the Municipal Government Plan

The Plan of Government of the Territory of the Municipality of Siziano in Tav. 3 8 “Map of administrative, landscape, and monumental constraints” identifies the 200 m buffer zones from potable water wells which are not compatible with the use of infiltration devices.

As can be seen from the following plan view, a well protection area partially overlaps a small portion of the site at the noarth-east corner. In this specific area, no infiltration works will be planned.

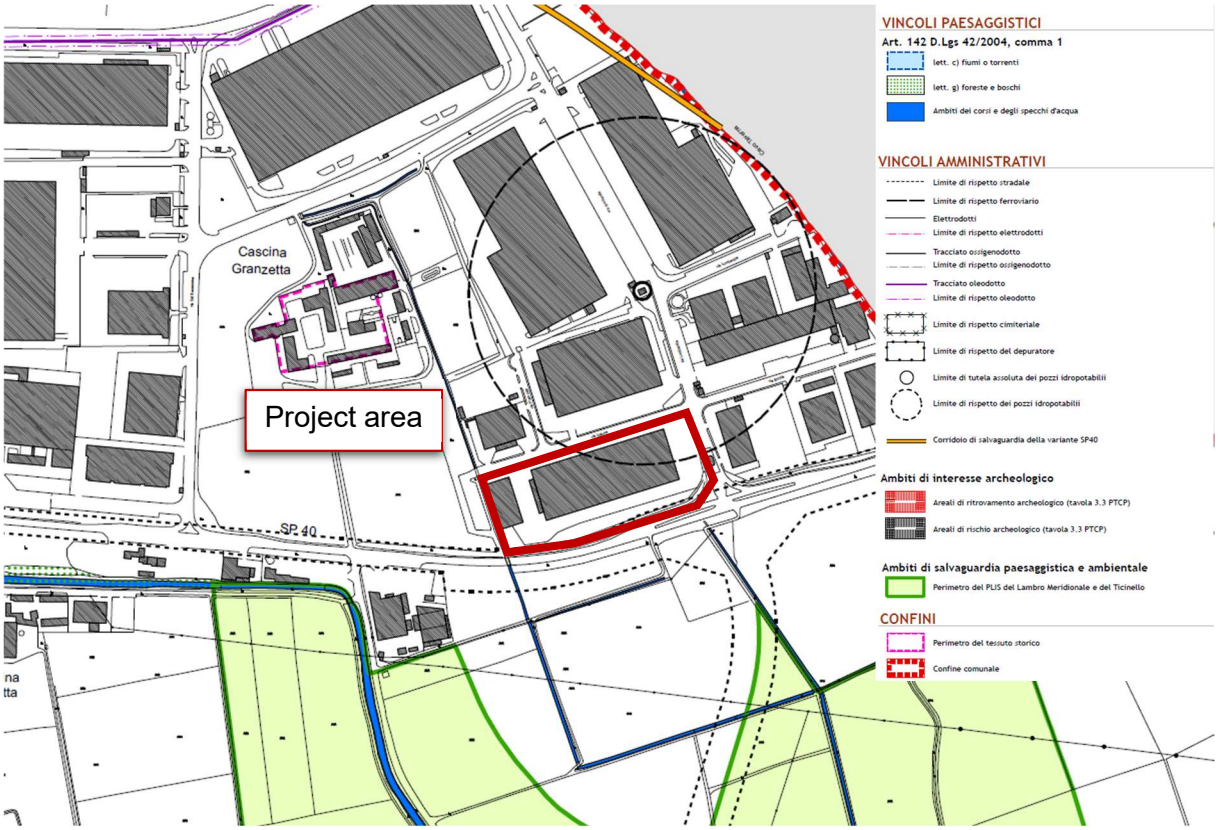

Figure 3.5: Map of administrative, landscape, and monumental constraints Tav.3 – PGT Siziano

3.4. Hydraulic permeability of soils

As shown in the cartographic extracts from the Plan for the Government of the Territory (PGT) of the Municipality of Siziano (Figure 3.6), the project area lies within a zone predominantly composed of poorly graded sand with silt and gravel. For this soil type, literature suggests a reasonable permeability range of approximately 10^{-4}–10^{-6} m/s.

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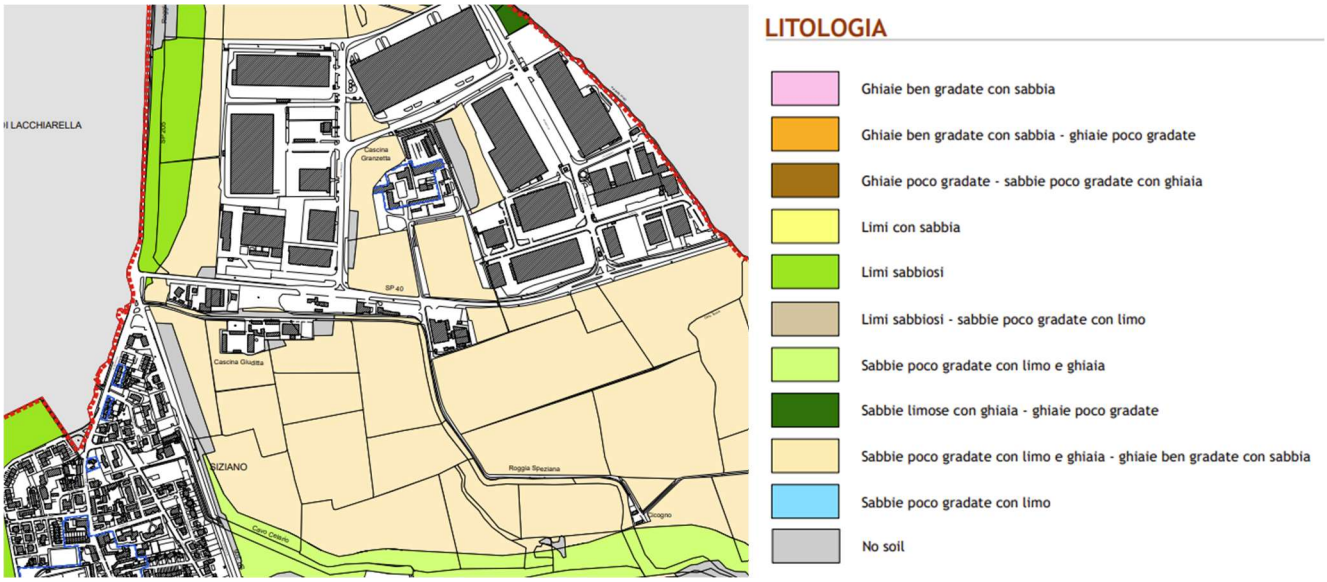

Figure 3.6: Litological map – PGT Siziano

During the execution of the continuous coring surveys, n.4 Lefranc permeability tests were carried out. Figure 3.7 shows a schematic representation of the test setup. In Table 3.2 are reported the reference values of the investigated depth intervals and the estimated permeability values. Based on these results. **The lowest coefficient of permeability identified by the Lefranc tests, equal to 2.4×10^{-5} m/s, was conservatively adopted.**

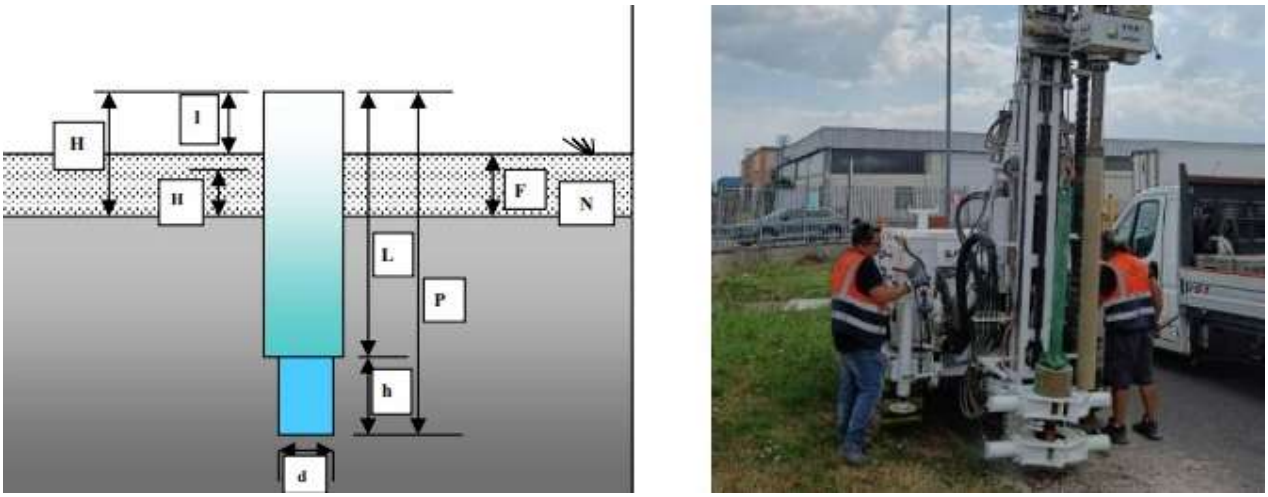

Figure 3.7: Permeability test – Lefranc method

Table 3.2: Results of permeability test – Lefranc method

SURVEY NUMBER	L [m]	P [m]	K [m/s]
Test 1	1.50	1.80	3.3E-05
Test 2	1.50	1.80	2.5E-05
Test 3	1.40	1.70	2.4E-05
Test 4	0.65	0.75	4.2E-05

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4. RISK ASSESSMENT OF SPILLS AND POTENTIALLY CONTAMINATED WATER

There areas within the project lay-out where accidental spills of pollutants could occur have been identified and are detailed below:

- areas designated for electrical generator installations and loading/unloading bay susceptible to oil and fuel spills (subject to regional regulation RR n. 4/2006 regarding first-flush);
- loading/unloading bay, although not subject to regional regulation RR n.4/2006 regarding first-flush, is nevertheless routed to the same drainage system as the gantry area due to site topography;
- areas traversed by vehicles (internal roads and yards), where oil and fuel spills are possible (not subject to regional regulation RR n. 4/2006 regarding first-flush).

To prevent accidental spills or contaminated water from reaching the detention tanks and the final water body receptor the following design measures have been implemented:

- electrical generator and loading/unloading bay areas:
    - installation of oil grit separators to treat precipitation up to 5 mm in height;
    - compliance with regional regulation RR n. 4/2006 requires treated flows to be directed to the public sewer;
    - only subsequent rainfall runoff from these areas will be directed to detention tanks.
- Internal roads and Yards:
    - continuous de-sanding and de-oiling treatment of surface runoff before it reaches the detention tanks.

These measures ensure that potential pollutants are effectively managed and contained, protecting soil and water quality within the project area.

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5. STORMWATER MANAGEMENT SITE CLASSIFICATION ACCORDING TO REGULATORY FRAMEWORK

5.1. Regional regulations 7/2017 criteria and methods for stormwater management compliance

According to Regional Regulation 7/2017 in Lombardy Region and subsequent amendments, interventions are classified based on class of intervention, project size, and runoff coefficient variation.

The project in question falls within the 1 to 10 hectares range, placing it in the "high potential imperviousness" class of interventions. Pursuant to Article 9 of R.R. 7/2017, this class requires the adoption of the “Detailed Procedure" calculation mode.

CLASSE DI INTERVENTO		SUPERFICIE INTERESSATA DALL'INTERVENTO	COEFFICIENTE DEFLUSSO MEDIO PONDERALE	MODALITÀ DI CALCOLO AMBITI TERRITORIALI (articolo 7)	
				Aree A, B	Aree C
0	Impermeabilizzazione potenziale qualsiasi	≤ 0,03 ha (≤ 300 mq)	qualsiasi	Requisiti minimi articolo 12 comma 1	
1	Impermeabilizzazione potenziale bassa	da > 0,03 a ≤ 0,1 ha (da > 300 mq a ≤ 1.000 mq)	≤ 0,4	Requisiti minimi articolo 12 comma 2	
2	Impermeabilizzazione potenziale media	da > 0,03 a ≤ 0,1 ha (da > 300 a ≤ 1.000 mq)	> 0,4	Metodo delle sole piogge (vedi articolo 11 e allegato G)	Requisiti minimi articolo 12 comma 2
		da > 0,1 a ≤ 1 ha (da > 1.000 a ≤ 10.000 mq)	qualsiasi		
		da > 1 a ≤ 10 ha (da > 10.000 a ≤ 100.000 mq)	≤ 0,4		
3	Impermeabilizzazione potenziale alta	da > 1 a ≤ 10 ha (da > 10.000 a ≤100.000 mq)	> 0,4	Procedura dettagliata (vedi articolo 11 e allegato G)	
		> 10 ha (> 100.000 mq)	qualsiasi		

Figure 5.1: Table 1 annexed to R.R. 7/2017 of Lombardy Region

Furthermore, R.R. 7/2017 specifies the maximum stormwater discharge values into receptors and minimum attenuation volumes, based on location within the regional territory, ensuring hydraulic and hydrological consistency amid land use changes.

The regional territory is categorised into three hydraulic criticality zones: A, B, and C.

According to Annex C of R.R. 7/2017, the project area falls into category B.

For this category, the discharge limit into receiving water bodies is set at 20 l/s/hectare of project area.

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6. HYDROLOGICAL AND HYDRAULIC ANALYSIS

The hydraulic-hydrological study was structured around the following key components:

- hydrological analysis:
  - identification of return periods;
  - determination of Height-Duration-Frequency (DDF) curves for various return periods;
  - identification of drainage areas and their runoff coefficients;
- preliminary hydraulic design of detention tanks:
  - construction of gross design hyetographs;
  - estimation of infiltration processes and identification of the net design hyetographs;
  - identification of the maximum discharge limit of each concrete underground tank;
  - simulation of the detention process and sizing of the detention basin using the detailed procedure for each concrete underground tank;
  - preliminary sizing of the discharge pipeline of each concrete underground detention tank;
- preliminary hydraulic design of detention and infiltration basin DP1:
  - preliminary calculations of the infiltration flowrate of the detention and infiltration basin;
  - simulation of the detention process and sizing of the detention basin using a simplified method;
- calculation of the emptying time of each detention tank and the detention basin.

6.1. Hydrological analysis

6.1.1. Identification of return period

The design rainfall event used in the hydrological calculations and hydraulic simulations is selected based on the concept of return period. The return period of an event refers to the average time between the occurrences of two successive events of equal or greater magnitude than a specified intensity value, or alternatively, the average time over which a specified intensity value is equalled or exceeded at least once.

The probability P_N that an event with a return period T will occur in N years is given by the following formula:

$$PN = 1 - \left(1 - \frac{1}{T}\right)^N$$

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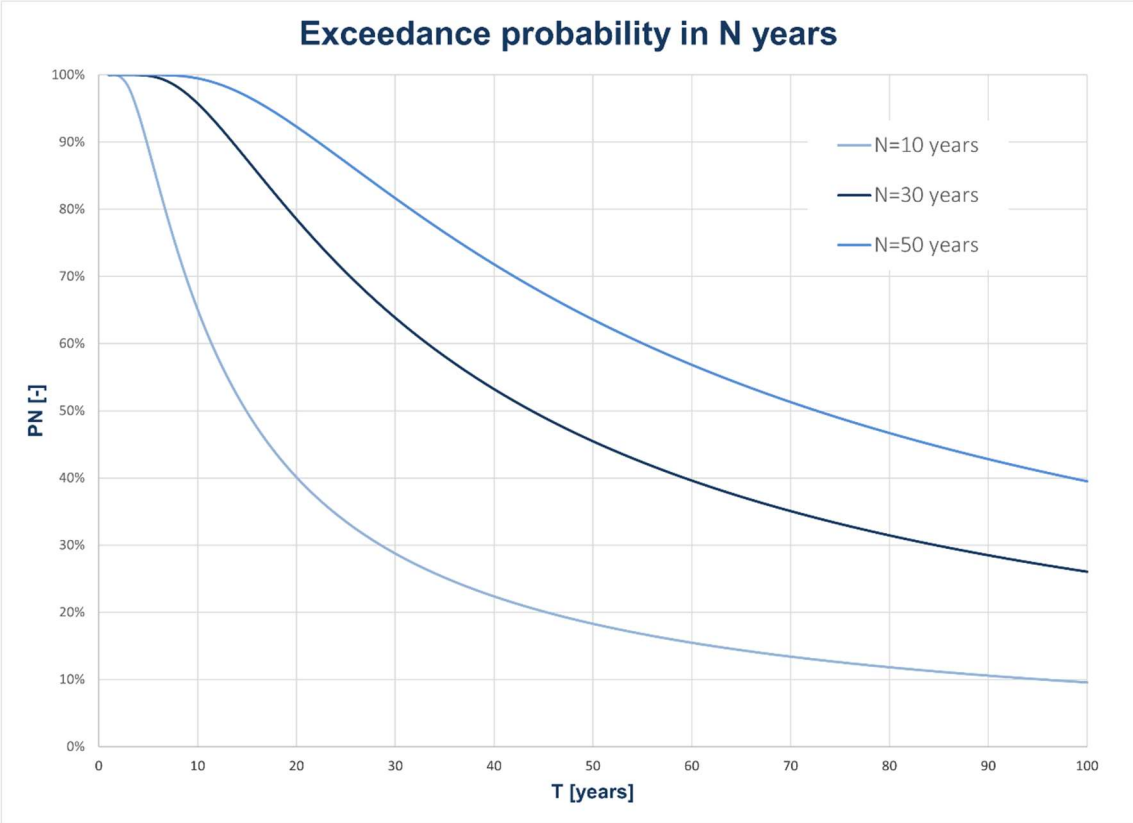

Figure 6.1: Probability of exceedance in N years

According to Article 11 of R.R. 7/2017, point 2, letter a, a return period of **50 years** is considered for sizing the detention basin, while a return period of **100 years** is used to verify the safety freeboards of the designed structure.

In subsequent design phases, the developer may request verification of safety freeboards even for scenarios with higher return periods.

6.1.2. Determination of DDF curves for various return periods

The height-duration-frequency (DDF) curves specify the rainfall height for a given duration, each referenced to a specified return period.

The formula utilized by ARPA Lombardy is presented below:

$$h(D) = a_1 \cdot w_T \cdot D^n$$

where:

- $h(D)$ represents the rainfall height for given duration D,
- a_1 and n are characteristic parameters of the rainfall probability curves, which are independent of return period and site-specific,
- w_T is a growth factor calculated based on site-specific parameters of the GEV (Generalized Extreme Value) statistical distribution:

$$w_T = \varepsilon + \frac{\alpha}{k}\left\{1 - \left[ln\left(\frac{T}{T-1}\right)^k\right]\right\}$$

The characteristic parameters for the DDF curves used to determine design precipitation conform to those provided by ARPA Lombardy for all locations within the region. These parameters are accessible via the ARPA Lombardy website, specifically through the link:

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http://idro.arpalombardia.it/pmapper4.0/map.phtml, covering various return periods and durations ranging from 1 to 24 hours.

For the specific area of interest, the DDF curve parameters are:

$$a_1 = 27.79$$

$$n = 0.2958$$

The GEV statistical distribution parameters are:

$$\alpha = 0.2912$$

$$\varepsilon = 0.8104$$

$$k = -0.069$$

In urban hydrology, where basins are typically small and runoff times are short, the focus is on intense rainfall events lasting less than an hour.

In the absence of alternative data, a value of 0.5 was adopted for the n parameter, as indicated in Annex G of R.R. 7/2017, while keeping the other parameters unchanged.

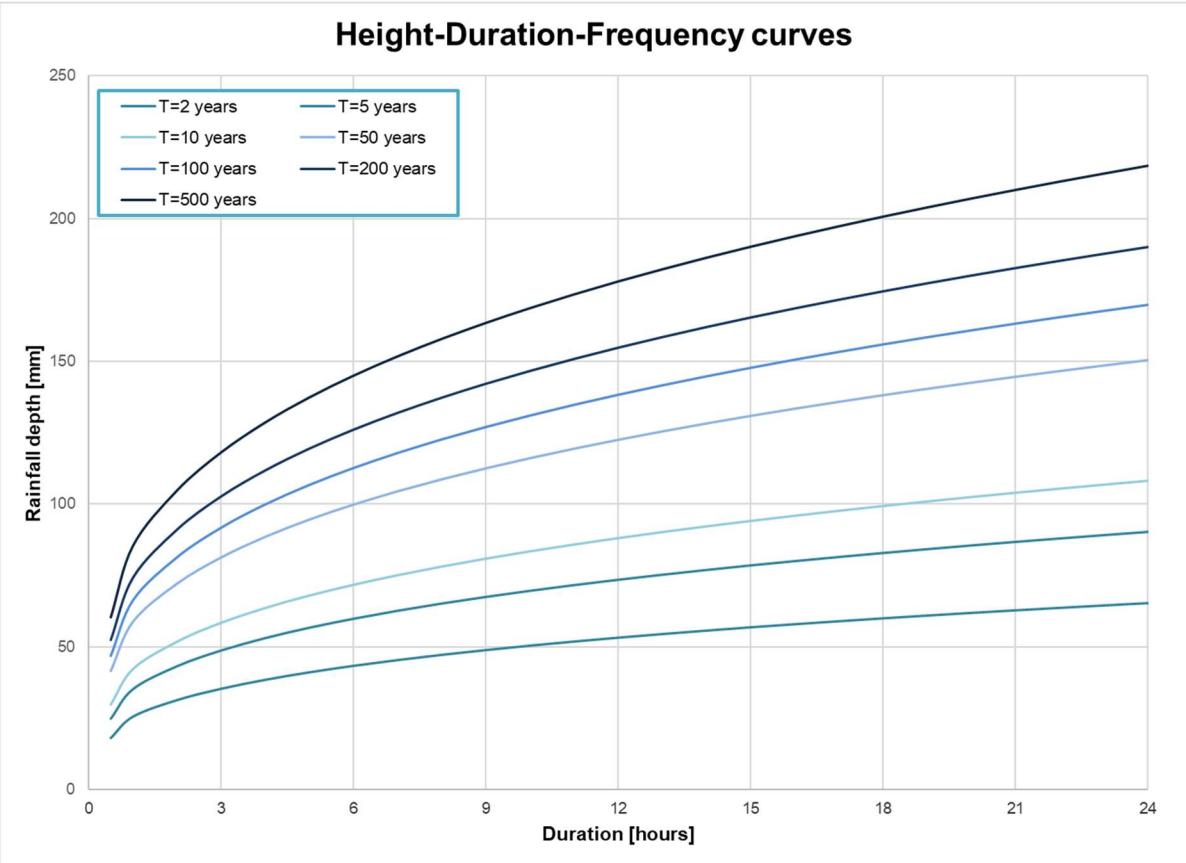

Figure 6.2: Height Duration Frequency curves (DDF)

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Table 6.1: Height Duration Frequency curves (DDF)

T [years]	2	5	10	50	100	200	500
W_t	0.92	1.27	1.52	2.11	2.39	2.67	3.07
d [hours]	**Rainfall depth h [mm]**						
0.5	18	25	30	42	47	53	60
1	26	35	42	59	66	74	85
2	31	43	52	72	81	91	105
3	35	49	58	81	92	103	118
4	38	53	64	89	100	112	129
5	41	57	68	95	107	120	137
6	43	60	72	100	113	126	145
7	45	63	75	104	118	132	152
8	47	65	78	109	123	137	158
9	49	68	81	113	127	142	163
10	50	70	83	116	131	147	169
11	52	72	86	119	135	151	173
12	53	74	88	123	138	155	178
13	55	75	90	125	142	159	182
14	56	77	92	128	145	162	186
15	57	79	94	131	148	165	190
16	58	80	96	133	151	169	194
17	59	82	98	136	153	172	197
18	60	83	99	138	156	175	201
19	61	84	101	140	158	177	204
20	62	86	102	143	161	180	207
21	63	87	104	145	163	183	210
22	64	88	105	147	166	185	213
23	65	89	107	149	168	188	216
24	65	90	108	150	170	190	218

Based on the considerations outlined above, the probability curves derived are as follow

- for durations longer than one hour:
  $h(D) = a_1 \, w_T \, D^n$
- for durations shorter than one hour:
  $h(D) = a_1 \, w_T \, D^{0.5}$

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6.1.3. Description of design drainage areas and runoff coefficients

To determine the design flow rate, the impermeable drainage area of the site is calculated.

The site area is divided into sub-areas based on their surface cover types; each assigned a specific permeability coefficient according to Regional Regulation 7/2017 and subsequent amendments of the Lombardy Region.

The effective rainfall, which represents the portion of precipitation contributing to the flood wave formation, is calculated using the percentage method. This method assumes a constant percentage of rainfall loss during the event. The runoff coefficient, denoted as φ, characterizes the event by comparing the volume of net precipitation (P_{net}) to total precipitation (P):

$$\varphi = \frac{P_{net}}{P}$$

Net rainfall is obtained by multiplying the synthetic design rainfall by the runoff coefficient φ, assuming uniform infiltration and runoff throughout the event.

The following tables provide a detailed breakdown of the design areas and their associated runoff coefficients for the catchment basins draining to detention tanks DT1 and DT2 and to the detention pond DP1.

Table 6.2: Area values and runoff coefficients for different cover types for the DT1 catchment basin

SURFACE TYPE	AREA [m²]	RUNOFF COEFFICIENT [φ]
Building covered area	3145	1
Asphalted roads & pedestrian walkways	5159	1
DT1	350	1
Permeable parking area	700	0.7
Photovoltaic panels	157	0.7
Total	**9512**	**0.97**

Table 6.3: Area values and runoff coefficients for different cover types for the DT2 catchment basin

SURFACE TYPE	AREA [m²]	RUNOFF COEFFICIENT [φ]
Building covered area	6772	1
Loading bay	454	1
Technical room	112	1
Asphalted roads & pedestrian walkways	2520	1
DT2	357	1
Permeable parking area	367	0.7
Total	**10582**	**0.99**

Table 6.4: Area values and runoff coefficients for different cover types for the DP1 catchment basin

SURFACE TYPE	AREA [m²]	RUNOFF COEFFICIENT [φ]
Green area	190	0.3
Total	**190**	**0.3**

The weighted average runoff coefficient is 0.97 for the DT1 detention tank, considering a total impermeable area of 0.93 hectares. For the DT2 tank, the weighted average runoff coefficient is 0.99 with a total impermeable area of 1.05 hectares. For the detention pond DP1 the average runoff coefficient is 0.3 with a total impermeable area of 0.0057 hectares.

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The weighted average runoff coefficient across the entire site is calculated to be 0.98, covering a total impermeable area of 1.98 hectares, as shown in the table below.

Table 6.5: Area values and runoff coefficients for different cover types for the entire site

SURFACE TYPE	AREA [m²]	RUNOFF COEFFICIENT [φ]
Impervious area	18869	1
Semi-permeable areas	1224	0.7
Permeable green areas	190	0.3
Total	**20284**	**0.98**

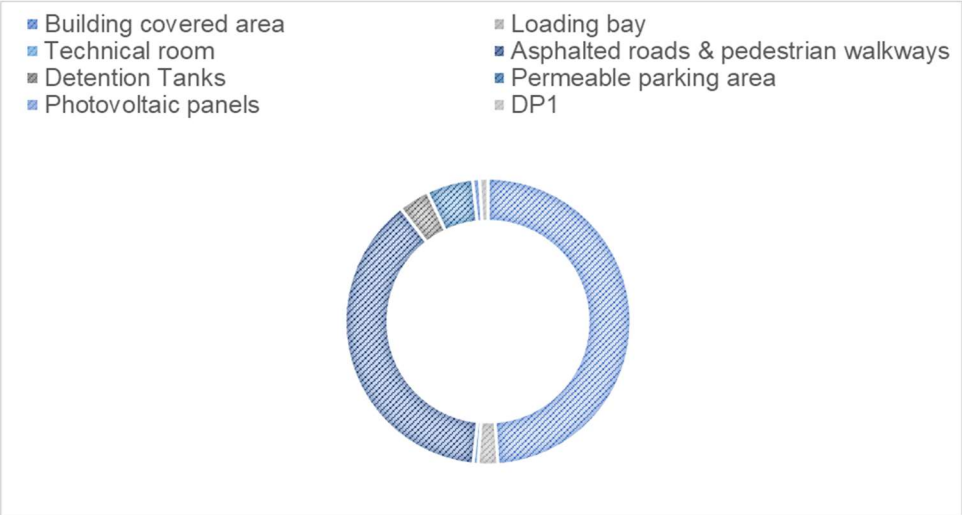

Figure 6.3: Land use distribution

It is reiterated that the external perimeter green areas have been excluded from the surface runoff analysis, as they will be kerbed and locally lowered so as not to contribute to site drainage. In subsequent design phases, this assumption shall be re-verified; if necessary, sustainable drainage measures (e.g., detention ponds, infiltration trenches) will be introduced to manage runoff within these areas.

6.2. Preliminary hydraulic design of detention tanks

6.2.1. Construction of the gross and net design hyetographs

A hyetograph represents the discrete time distribution of precipitation intensity or height during a storm event.

In this project, the theoretical model for hyetographs is rectangular. The rectangular hyetograph assumes constant precipitation intensity for a fixed event duration and return period.

To determine rainfall durations corresponding to the design return time (**50 years**) and safety freeboard verification (**100 years**), a range of durations was analysed.

During the pre-dimensioning stage, numerous precipitation durations were evaluated to identify critical scenarios using a simplified rainfall-only method.

In the sizing phase, emphasis was placed on durations around the critical duration that maximizes attenuation volume.

This detailed procedure was performed for the two detention tank in design (**DT1**, **DT2**)

The gross precipitation intensities considered for these durations are depicted in the following figures:

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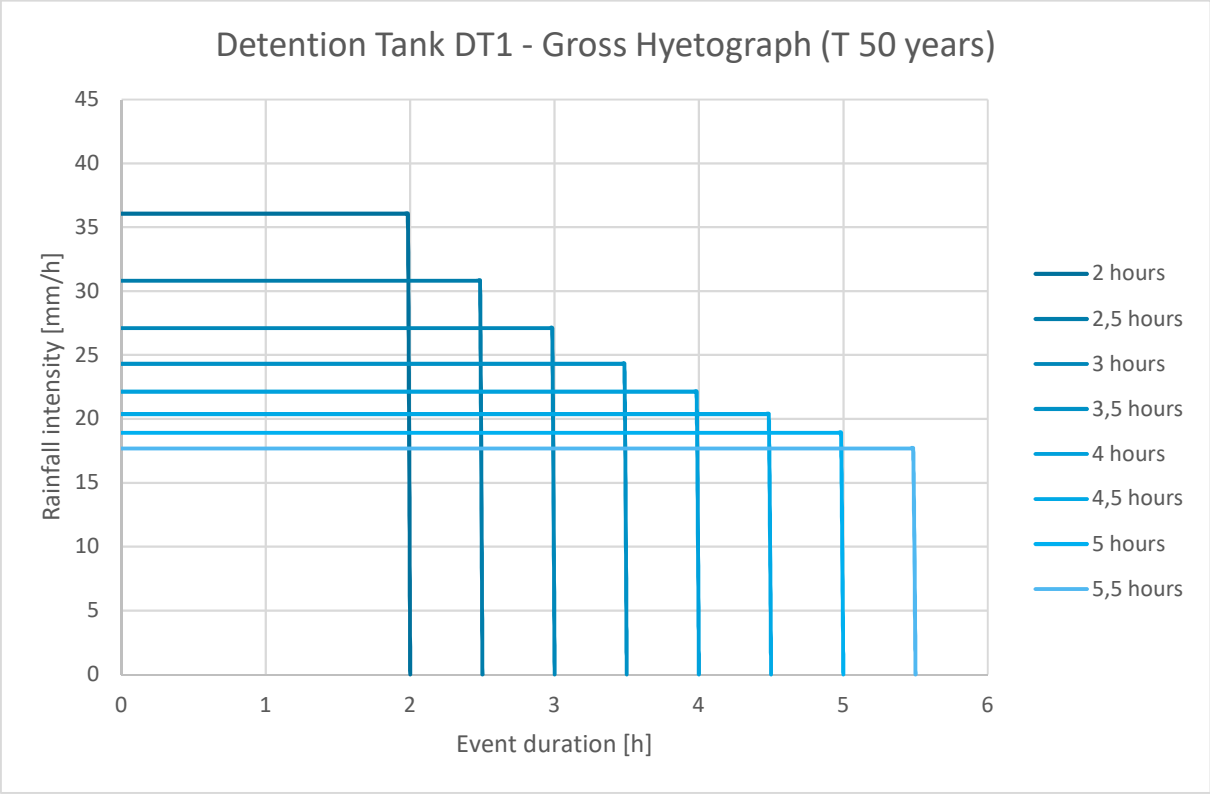

Figure 6.4: Gross rainfall intensities for the DT1 detention tank (return period 50 years)

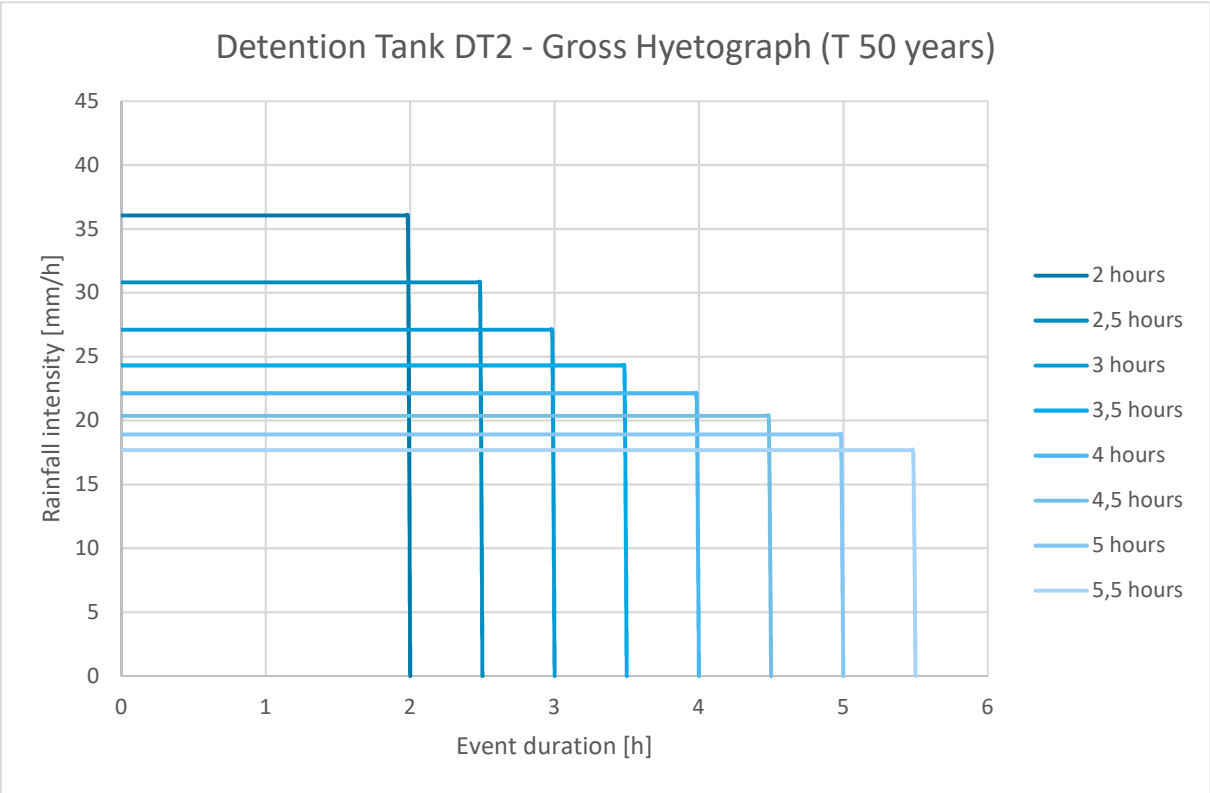

Figure 6.5: Gross rainfall intensities for the DT2 detention tank (return period 50 years)

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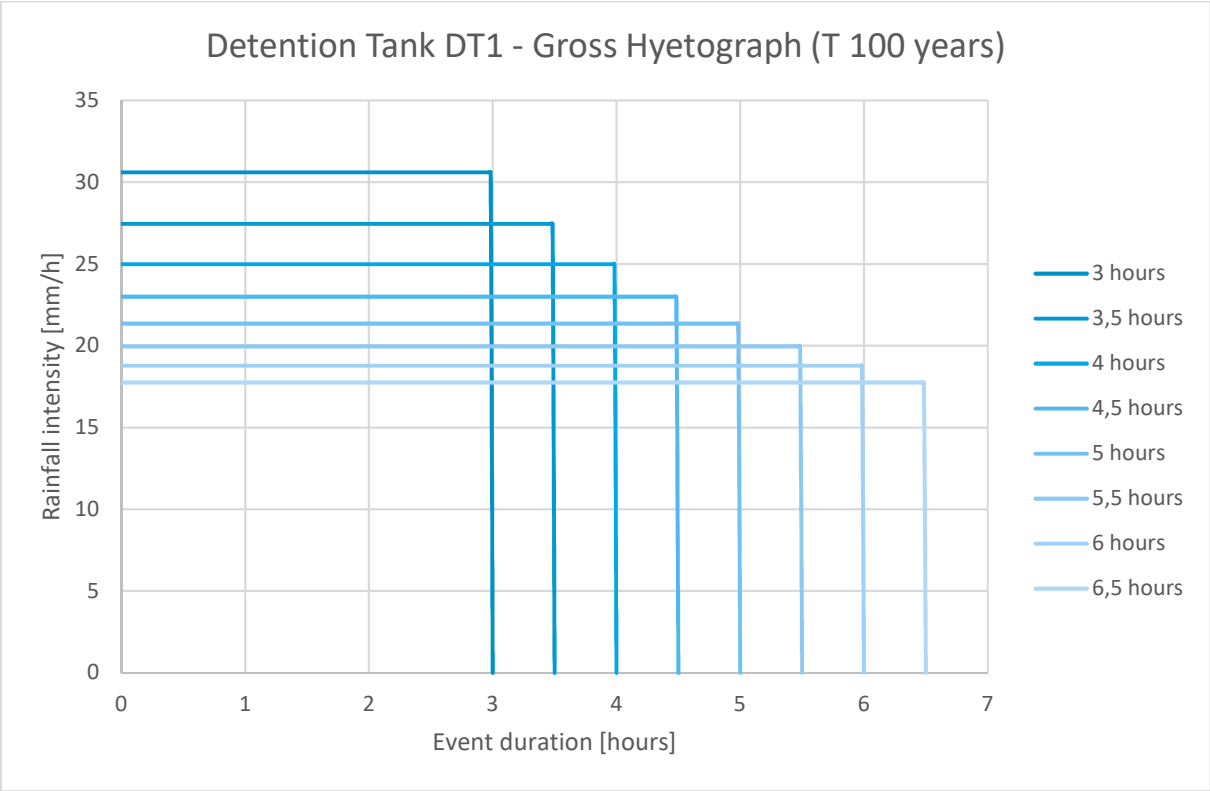

Figure 6.6: Gross rainfall intensities for the DT1 detention tank (return period 100 years)

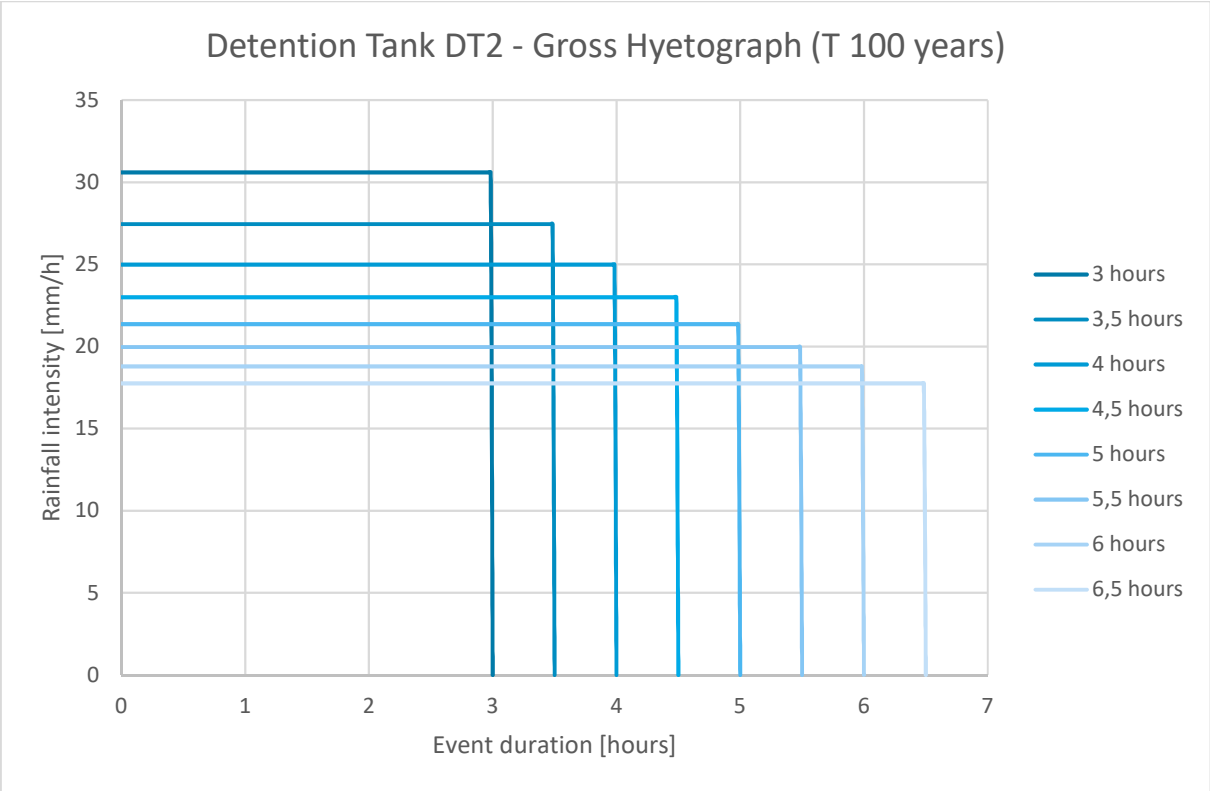

Figure 6.7: Gross rainfall intensities for the DT2 detention tank (return period 100 years)

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Hydrological losses were calculated using the percentage method described in Section 6.1.3, with an inflow coefficient of 0.97 for detention basin DT1 and 0.99 for detention basin DT2.

Table 6.6: Runoff coefficient for each Concrete Detention Tank

ID TANK	AVERAGE RUNOFF COEFFICIENT [-]
DT1	0.97
DT2	0.99

Net rainfall intensities associated with the hyetographs are shown in the following graphs.

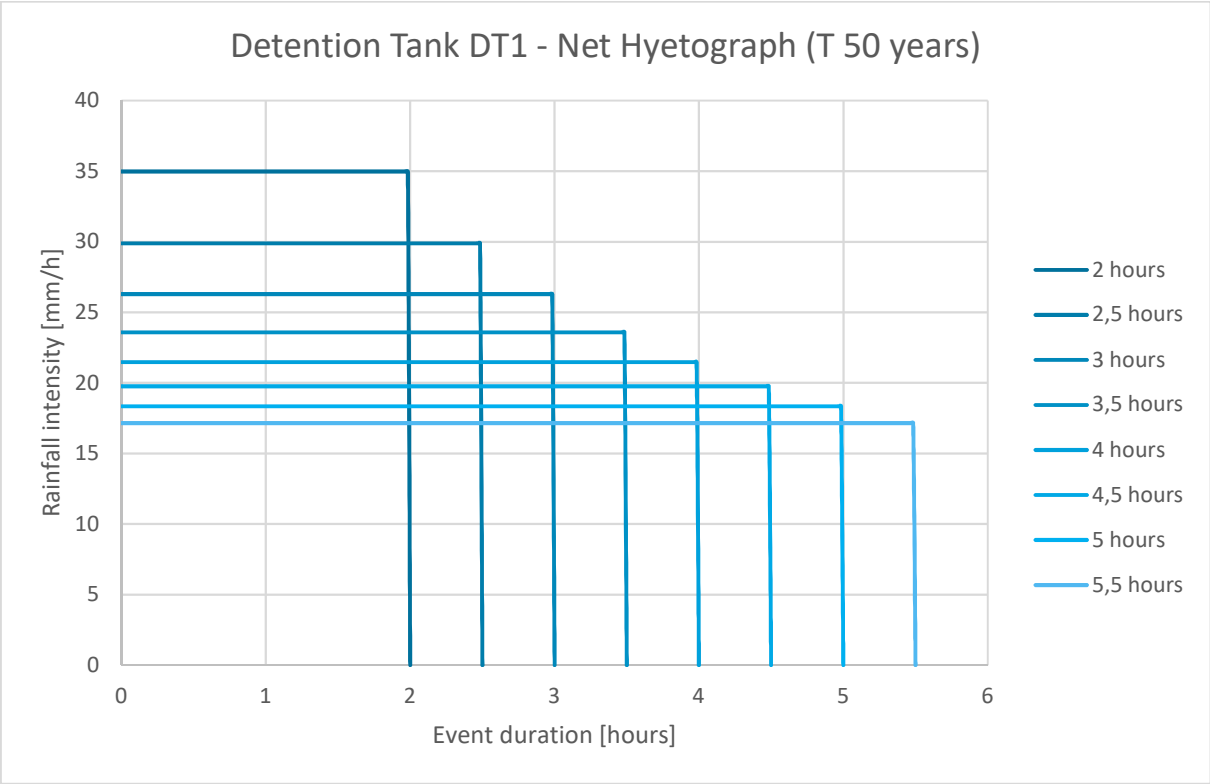

Figure 6.8: Net rainfall intensities for the DT1 detention tank (return period 50 years)

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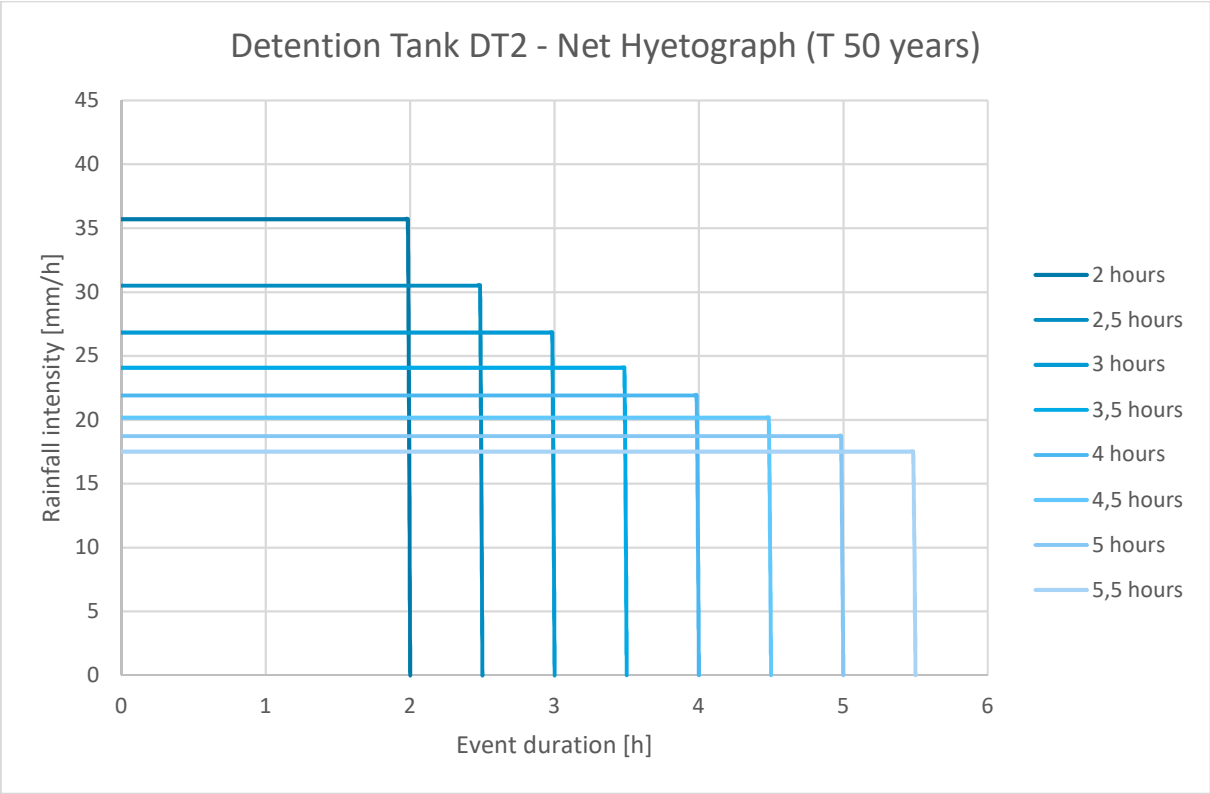

Figure 6.9: Net rainfall intensities for the DT2 detention tank (return period 50 years)

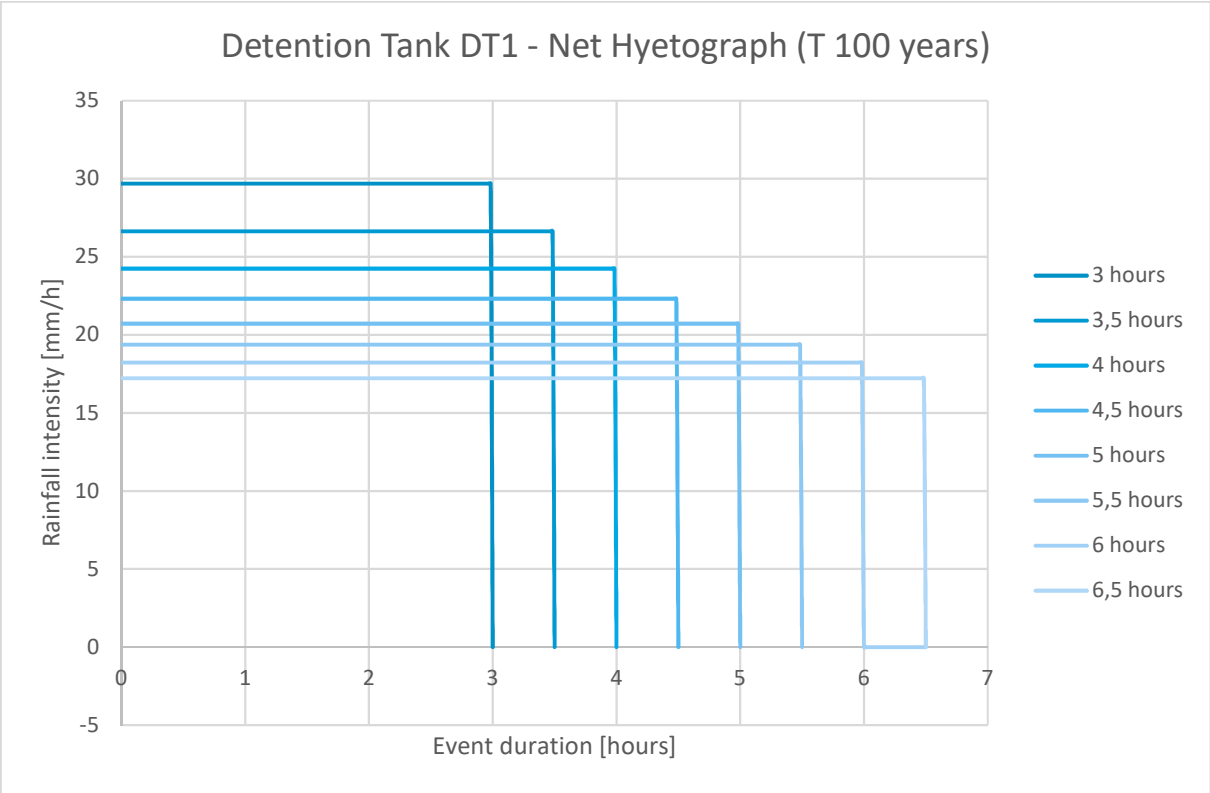

Figure 6.10: Net rainfall intensities for the DT1 detention tank (return period 100 years)

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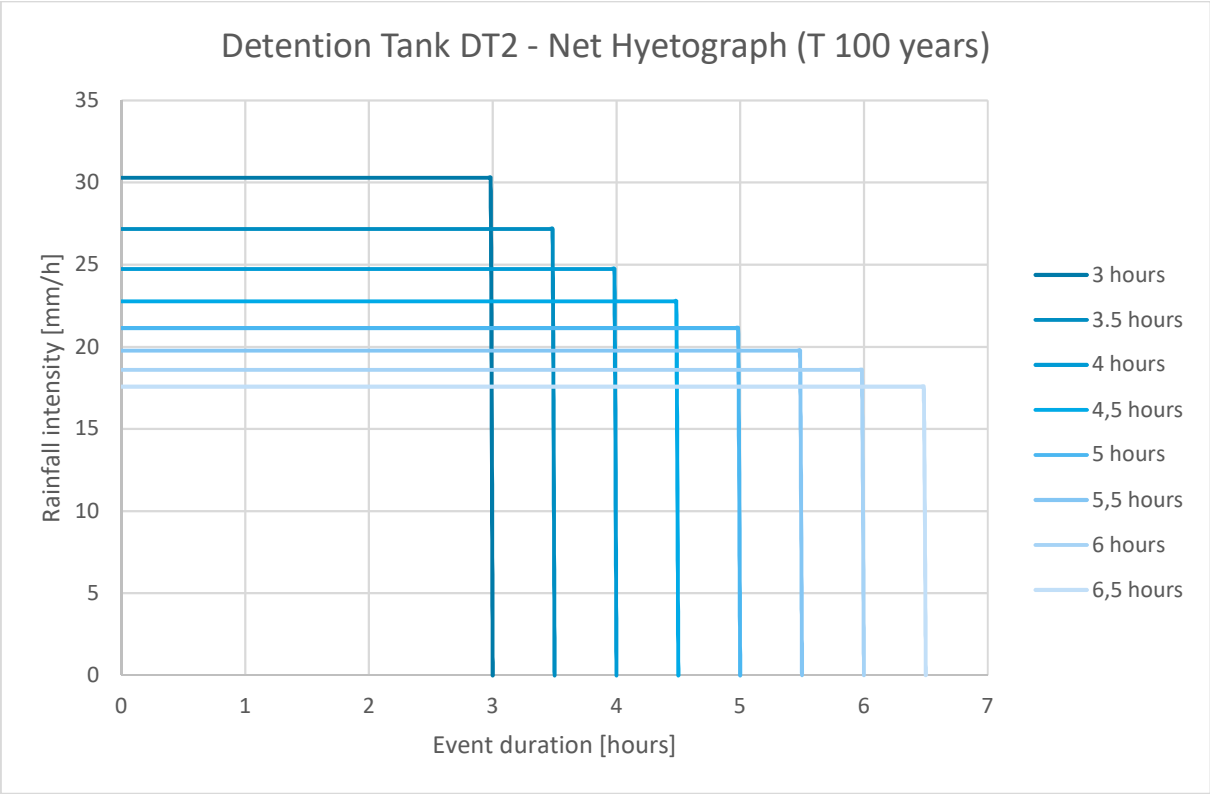

Figure 6.11: Net rainfall intensities for the DT2 detention tank (return period 100 years)

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6.2.2. Identification of the maximum discharge limit of each concrete underground tank

In accordance with Regional Regulation 7/2017 and its subsequent amendments and additions, the maximum permissible flow rate for discharge is 20 l/s per hectare of intervention area.

In accordance with the areas indicated in paragraph 6.1.3, the maximum permissible flow rate for discharge for each of the reservoirs is shown below.

Table 6.7: Maximum allowed discharge from each tank in accordance with Regional Regulation 7/2017

ID TANK	LIMIT FLOW RATE AT DISCHARGE [l/s]
DT1	19.0
DT2	21.1
Total	**40.1**

The Roggia Cariola—identified as the final receiving water body —is managed by the Naviglio Olona Consortium, which has imposed a maximum admissible discharge of 30 l/s due to hydraulic compatibility with the channel cross-section. Accordingly, the following outlet flow limits are adopted.

Table 6.8: Maximum allowed discharge from each tank in accordance with 30 l/s limit imposed by Consorzio Naviglio Olona

ID BASIN	LIMIT FLOW RATE AT DISCHARGE [l/s]
DT1	14.0
DT2	16.0
Total	**30.0**

The discharge point in the Roggia Cariola (S1) has the following coordinates relative to the WGS 84 32 N (EPSG 32632) design reference system:

Table 6.9: Location of design discharge S1

ID	X [m] WGS 84 UTM 32	Y [m] WGS 84 UTM 32
S1	516600	5019079

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6.2.3. Planimetric location of concrete underground detention tanks

There are two concrete underground detention tanks in design, identified as **DT1, DT2.**

The **DT1** tank, located in the south-west sector of the site, receives runoff from the streets and yards in the north, north-west, and south sectors, as well as roof drainage from the south façade of the main building downspouts.

The **DT2** tank, located in the south-east sector, receives runoff from the north-east and east streets and yards, roof drainage from the north façade downspouts, and the potentially oil-contaminated runoff from the generator area and the loading/unloading bay.

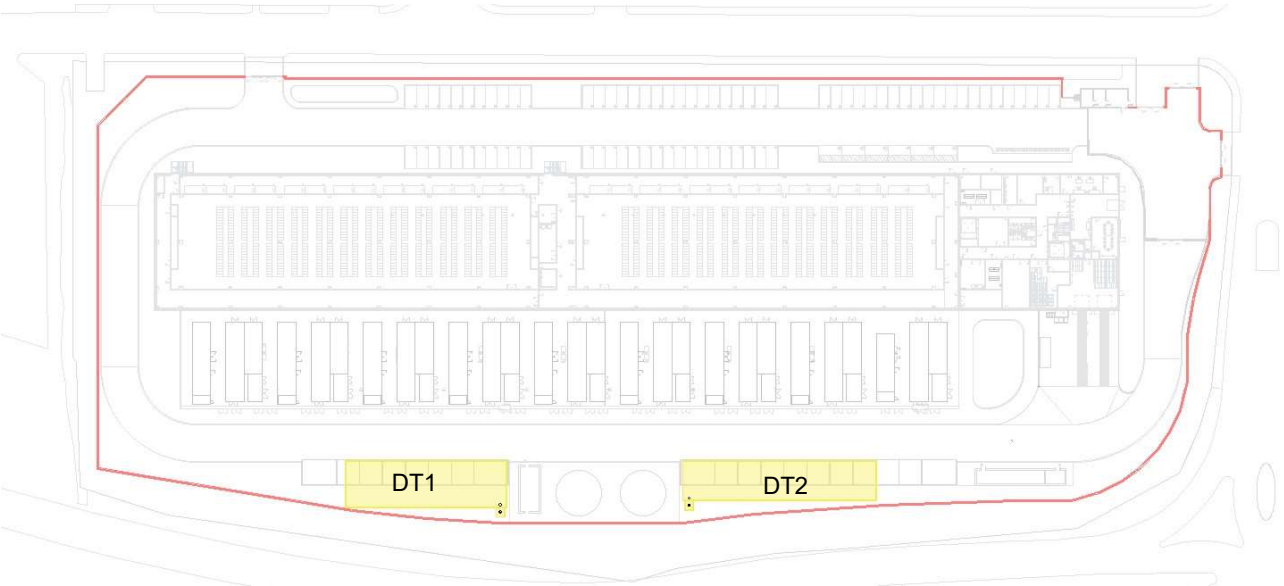

Figure 6.12: Layouts plan of the detention tanks DT1 and DT2

6.2.4. Detention underground tanks sizing

For the calculation of detention volumes, impermeable surfaces as defined in chapter 6.1.3 and flow rates from chapter 6.2.2 were considered, summarised in the table below.

Table 6.10: Outlet flow rate and impermeable drainage area influencing the two designed detention basins

ID DETENTION BASIN	FLOW RATE AT DISCHARGE [l/S]	IMPERMEABLE DRAINAGE AREA [ha]
DT1	14	0.93
DT2	16	1.05

The effectiveness of a static-type reservoir's attenuation process is influenced by its maximum containment volume, geometry, and outlet characteristics, which in this case are represented by the pumping system discharging to the Roggia Cariola.

The mathematical description of the attenuation process over time t follows these equations:

- differential continuity equation:

$$Q_e(t) - Q_u(t) = \frac{dW(t)}{dt}$$

- outflow law:

$$Q_u = Q_u\,[h(t)]$$

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- reservoir curve, expressing the geometric link between the volume and the water depth in the detention basin:

$$W = W\,[h(t)]$$

where $Q_e(t)$ represents the inflow rate, $Q_u(t)$ represents the outflow rate and $W(t)$ is the detention volume as a function of water depth $h(t)$.

The detailed theoretical model for determining the detention volume operates under the following assumptions:

- incoming precipitation matches the hydrologic flow rate derived from synthetic hyetographs presented in Chapter 6.2;
- utilization of a linear time-area curve;
- application of an inflow-outflow model based on the kinematic method;
- maintenance of a constant outflow rate equivalent to the maximum discharge limit detailed in Chapter 6.2.2;

The calculation was performed with different precipitation durations for both the DT1 and DT2 detention volumes. Specifically, the maximum attenuation volume for the **DT1** detention tank occurs for the duration of 5.0 hours for a **50-year** return period and 6.0 hours for a **100-year** return period while for the **DT2** detention tank, the maximum attenuation volume occurs for the duration of 5.0 hours for a **50-year** return period and 6.0 hours for a **100-year** return period.

Parameters used in the calculation for both basins are summarized in the following table.

Table 6.11: Parameters for calculating DT1 detention volume

DETENTION VOLUME CALCULATION	
Impermeable drainage surface [ha]	0.93
Weighted average runoff coefficient [-]	0.97
Design volume (50 years) [m^3]	613
Control volume (100 years) [m^3]	730
Outlet flow rate [l/s]	14

Table 6.12: Parameters for calculating DT2 detention volume

DETENTION VOLUME CALCULATION	
Impermeable drainage surface [ha]	1.05
Weighted average runoff coefficient [-]	0.99
Design volume (50 years) [m^3]	697
Control volume (100 years) [m^3]	830
Outlet flow rate [l/s]	16

The geometric design of the detention basins was specifically sized to achieve the following:

- ensuring a volume greater than the design volume necessary for the **50-year** return period event, situated below the lowest pipelines invert level at the inlet to each detention tank;

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- matching the design volume required for the 100-year return period event, situated below the centerline of the same pipeline.

6.2.5. Sizing of pressure pipeline from detention tanks

A total of two pressure discharge pipelines are planned in the project, each connected to a pumping station that empties tanks DT1 and DT2. The two discharge lines merge into a common stilling manhole from which the combined flow continues by gravity to the surface water body Roggia Cariola at outfall S1.

Within each of the lifting stations, it is planned to install two pumps, each capable of lifting the design flowrate one for normal operation and one for reserve to guarantee the mechanical redundancy of the discharge.

Furthermore, each of the pumps will have to be connected both to the BSM system and to the power supply from housing GENSET to guarantee electrical redundancy under all operating conditions.

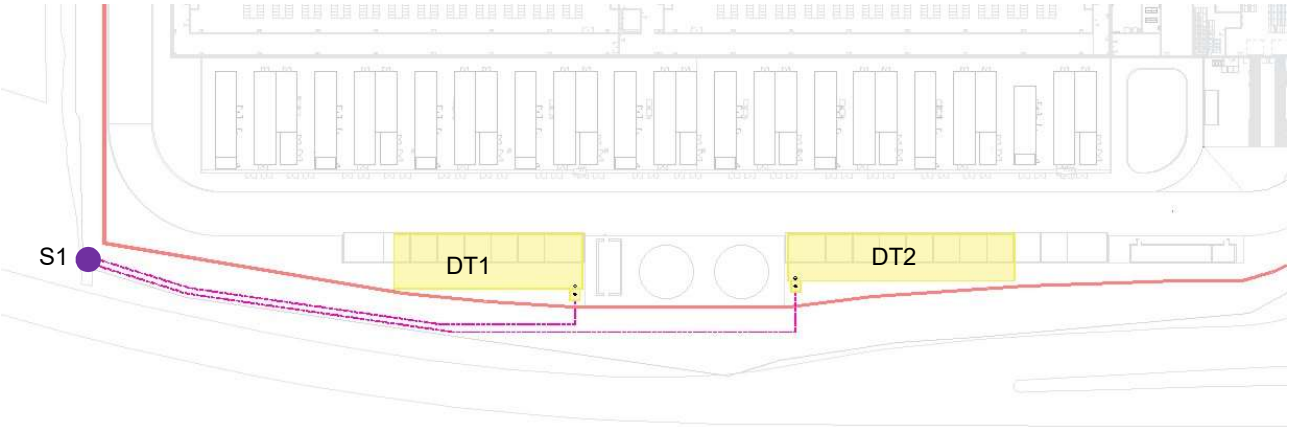

Figure 6.13: Discharges of DT1 and DT2 detention tanks into Roggia Cariola at outfall S1

Within each of the lifting stations, it is planned to install two pumps, each capable of lifting the design flowrate one for normal operation and one for reserve to guarantee the mechanical redundancy of the discharge.

At the current stage, no topographic survey is available for the receiving watercourse (Roggia Cariola) nor the absolute elevations of tanks DT1 and DT2. Consequently, the sizing of the of the delivery pipes and the selection of the pumps shall be finalized in the next design phase, once the required level data are available. The coordinates of discharge point S1 are the following:

Table 6.13: Location of design discharge S1

ID	X [m] WGS 84 UTM 32	Y [m] WGS 84 UTM 32
S1	516600	5019079

6.3. Preliminary hydraulic design of detention and infiltration pond DP1

6.3.1. Preliminary calculation of the infiltration flow rate of the detention and infiltration pond DP1

In this preliminary sizing phase, the infiltration rate will be determined using Darcy's Law:

$$Q = k \cdot A = 4.4\ l/s$$

Where:

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- Q is the infiltration flow-rate;
- k is the hydraulic permeability hypothesised in Chapter 3.4 (2.4×10^{-5} m/s);
- A is the surface of the detention pond bottom (140 m^2).

Figure 6.14: Layouts plan of the detention basin DP1

6.3.2. Preliminary detention and infiltration basin DP1 sizing

In this preliminary phase the detention volume was calculated with a simplified method based on the following assumptions:

- Constant inflow rate;
- Constant outflow rate.

Below is theoretical overview underlying the sizing process, aimed at determining the required storage volume for stormwater detention during peak rainfall events, in relation to the return period and the maximum allowable discharge rate.

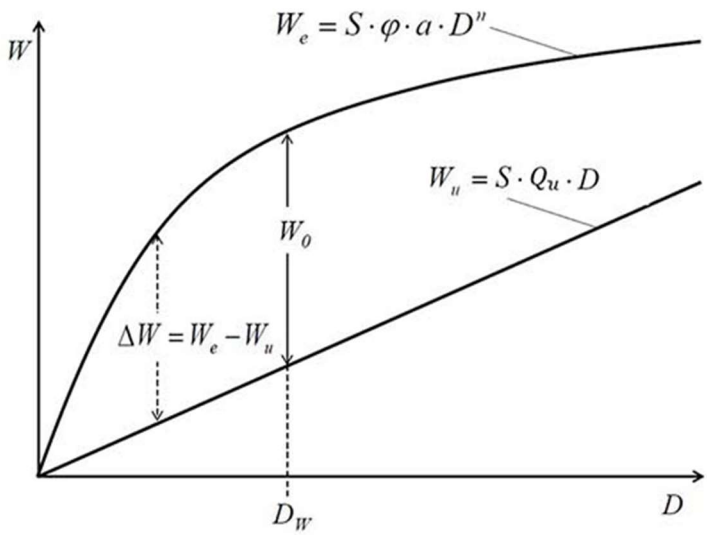

Figure 6.15: Detention Scheme based on the "sole piogge" method

This simplified method, through the analytical formulation presented below, allows for the calculation of the critical duration D_w of the event that maximizes storage demand, and consequently, the required detention volume W_0.

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$$D_w = \left(\frac{Q_u}{2{,}78 \cdot S \cdot \varphi \cdot a \cdot n}\right)^{\frac{1}{n-1}}$$

$$W_0 = 10 \cdot S \cdot \varphi \cdot a \cdot D_w^n - 3{,}6 \cdot Q_u \cdot D_w$$

Where:

W_0 = detention volume [m³];

S = impervious drainage area [m²];

a = scale parameter [mm/hⁿ];

n = shape coefficient [-];

φ = runoff coefficient [-];

D_w = critical duration [h];

Q_u = outflow rate (infiltration) [l/s];

The following table summarizes the minimum detention volume calculated for return periods ranging from 2 to 100 years for the detention and infiltration pond DP1 and for each return period, the average expected depth was calculated for the assumed basin bottom surface.

Table 6.14: Detention volume calculation for DP1

T [years]	S [ha]	$Q_{u,lim}$ [l/s]	a [mm/h]	n	φ	D_w [h]	W_0 [m³]	depth [cm]
2	0.014	4.4	26	0.5	0.3	0.002	0.03	0.0
5	0.014	4.4	35	0.5	0.3	0.004	0.06	0.0
10	0.014	4.4	42	0.5	0.3	0.006	0.09	0.1
20	0.014	4.4	49	0.5	0.3	0.008	0.13	0.1
50	0.014	4.4	59	0.5	0.3	0.011	0.18	0.1
100	0.014	4.4	66	0.5	0.3	0.014	0.23	0.2

6.4. Emptying time

Below are the calculations for the drainage time of the designed catchments for both the sizing return period (50 years) and the verification return period (100 years).

The detention tanks and basin empty within less than 24-48 hours in all assessed scenarios.

Table 6.15: DT1 detention tank emptying times

EMPTYING TIMES			
T [years]	**Q_u [l/s]**	**V [m³]**	**t_e [h]**
50	14	613	12.2
100	14	730	14.5

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Table 6.16: DT2 detention tank emptying times

EMPTYING TIMES			
T [years]	**Q_u [l/s]**	**V [m^3]**	**t_e [h]**
50	16	697	12.1
100	16	830	14.4

Table 6.17: DP1 detention pond emptying times

EMPTYING TIMES			
T [years]	**$Q_{u,inf}$ [l/s]**	**V [m^3]**	**t_e [h]**
50	4.4	0.18	0.01
100	4.4	0.23	0.01

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7. CONCLUSIONS

The purpose of this document is to describe the design and verification of the Stormwater Management works for the project in Siziano.

This document constitutes the preliminary study for defining the stormwater management strategy and does not represent the stormwater management project in accordance with R.R. no.7 of 23 November 2017 of Lombardy Region as amended and supplemented.

The project team approached the design of stormwater management systems in an integrated manner from the earliest stages, enabling the implementation of sustainable drainage solutions that comply with European policies as well as national and regional regulations on hydraulic risk, sustainability, and climate change adaptation and mitigation.

A multi-criteria analysis was carried out prior to the design phase of these systems, considering the following factors:

- possible infiltration constraints from the Municipal Government Plan,
- site topography,
- creation of a balanced master plan in terms of green and impermeable areas,
- assessment of areas at risk of spillage or subject to the first-flush regulation;

The project presents many constraints from an environmental point of view, which are discussed in detail in chapter 3, the most relevant of which are poor soil quality and low water table depth. For this reason, it was decided to use underground concrete tanks to hold impermeable and covered surfaces.

The project includes separate collection networks for runoff water from the yards, which undergo dedicated pre-treatment for sand and oil removal, and rainwater from downspouts. For detailed sizing information, refer to the report MIL07-DBA-EX-XX-CA-C-52001. Both types of water are directed towards concrete underground detention tanks.

To optimize the drainage network – minimizing pipe runs, diameters, and excavation depths – the layout adopts two underground detention tanks (**DT1** and **DT2**). DT1, located in the south-west sector of the site, collects runoff from streets and yards in the north-west, west, and south areas, as well as roof flows from downspouts along the main building's south façade. DT2, located in the south-east sector, collects runoff from streets afnd yards in the north-east and east areas, roof flows from downspouts along the north façade, and runoff from areas subject to the regional first-flush regulation (RR 04/2006). For the sizing of these works please refer to Chapter 6.

To make the green areas hydraulically independent, they will be kerbed and locally lowered to prevent runoff from draining into the site. A detention pond (**DP1**) is planned within the internal green area.

In the subsequent design phase, the inclusion of infiltration works at the bottom of DP1 will be assessed. Once the existing and design elevations of the external perimeter green areas are defined, the need for additional detention basins or other NBS will also be evaluated.

The positioning and preliminary sizing of DP1 will be constrained by:

- the permeability of the soils forming the project embankment;
- any environmental constraints related to soil quality;
- interference with planned underground utilities;
- coordination with the landscape design;
- potential interference with the groundwater table.

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These same constraints will also apply to any additional detention ponds or NBS that may be designed in the subsequent phase. For the sizing of this work please refer to Chapter 6.3.

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